Modelling of moment transmitting beam-to-column timber connections accounting for frictional transmission

M. Basterrechea-Arévalo a, J.M. Cabrero a,b,∗, B. Iraola a, R. Goñi b
a Universidad de Navarra, Wood Chair, Department of Building Construction, Services and Structures, 31009 Pamplona, Spain
b Universidad de Navarra, Department of Building Construction, Services and Structures, 31009 Pamplona, Spain

A R T I C L E   I N F O

Keywords:
Dowel
Connection
Timber
Moment
Finite element modelling
Contact pressure
Load distribution

A B S T R A C T

The development of accurate structural modelling techniques is required to promote the use of timber as a renewable alternative to other structural materials. Due to their remarkable influence on the global behaviour of a timber structure, an accurate description of the performance of structural connections is needed. Particularly in the case of moment transmitting beam-to-column connections with dowel-type fasteners, such properties are difficult to obtain experimentally. This paper develops a finite element (FE) model that simulates the behaviour of these connections under quasi-static loading, with a focus on the estimation of rotational stiffness and load distribution among the dowels. The model is validated against short-term laboratory tests. Resulting friction between timber members due to the installation procedure must be considered, as it greatly influences the rotational response. Besides, non-linear behaviour of timber and a softened contact parameter have been implemented, and their influence on the FE model validation process is demonstrated.

1. Introduction

The increasing use of timber structures is inspired both by the concern about sustainability and by recent policies which strengthen the proliferation of forests and the use of wood in construction [1].

Mechanical connections with steel dowel-type fasteners, such as dowels or bolts, are among the most used connections in architecture and civil engineering, due to their simplicity of manufacturing and assembling, both in historical and modern constructions [2,3]. Although typical assumptions consider timber connections with mechanical fasteners as fully pinned or rigid, several works have pointed out the need to adequately assess their stiffness for a proper evaluation of structural requirements (i.e. stability, serviceability) [4,5].

The structural response of moment transmitting connections, which are increasingly used in portal frames [6], is actually semi-rigid [7]. Their response may be characterized by moment–rotation angle curves [8]. The transfer of bending moment is actually performed by the lateral loading of dowel-type fasteners, which may transfer the load in different directions in relation to the timber members. Therefore, improving the characterization of such moment transmitting connections by adequate modelling strategies would allow to fully comprehend their structural response and thus increase their use.

Leško [9] summarized the most common moment transmitting connections, from the classical ones to the latest developments, and emphasized the necessity of universal experimentally validated 3D models to be used in the design of these type of connections. Some analytical models have been recently developed, capable of predicting the stiffness of the connection, considering assumptions related with the material rigidity and slip behaviour of dowels [10]. However, most of the proposed models for timber connections focus on axially loaded connections [10,11], as do mostly the current design rules, such as the Eurocode 5 [12]. There is no background on how to obtain the stiffness, and the load distribution among fasteners is based on simplified analytical models [13,14].

Several investigations have been carried out in order to determine the mechanical behaviour of moment transmitting connections. The existing research works focus mostly on connections with steel plates and glued-laminated timber members. Mori et al. [15] investigated the performance of beam-to-column connections with special connectors, concluding that, as expected, the overall moment resistance and stiffness of the connection increased as the connection’s depth did, while the ductility decreased. Dourado et al. [16] studied the influence of the spacing among dowels on the initial stiffness and moment-carrying capacity of a moment resisting connection formed by two wood (pine) members and a thick metal plate duly fastened with four steel dowels, in an L-shaped configuration. The developed FE model, which incorporated a self-developed mixed-mode cohesive zone model, was validated with the experimental load–displacement curves and the resultant damage profile in the vicinity area of the steel dowels.

∗ Corresponding author.
E-mail address: jcabero@unav.es (J.M. Cabrero).

https://doi.org/10.1016/j.engstruct.2021.113122
Received 12 March 2021; Received in revised form 9 August 2021; Accepted 31 August 2021
Available online 9 September 2021
0141-0296/© 2021 The Authors. Published by Elsevier Ltd. This is an open access article under the CC BY-NC-ND license
Awaludin et al. [17] investigated the effect of pretension in bolts on hysteretic responses and ultimate properties of moment transmitting timber connections with steel side plates. A great increase of initial stiffness in the prestressed connections was noticed, but slight increments in ductility coefficient and ultimate moment resistance. Awaludin et al. [18] also evaluated the cyclic behaviour of moment transmitting timber connections with high-strength dowels, analysing some locally reinforced connections as well. In addition, some connections with normal-strength dowels were tested using the same loading protocol for comparison. The effect of dowel-grade on both moment resistance and hysteretic damping was found to be only significant in the case of reinforced connections.

Bader et al. [19] studied the behaviour of individual dowels in multi-dowel connections loaded by a bending moment. For this purpose, double-shear, steel-to-timber connections with nine steel dowels arranged in circular and square patterns and with different dowel diameters were tested in four-point bending. The connections were partly reinforced to achieve a ductile behaviour, even though it had no influence on the global load–deformation behaviour. The moment capacity and rotational stiffness were higher for the square pattern. They classified two different types of deformations in the dowel: crushing deformation, defined as the relative deformation of the individual dowels at the steel plate with respect to their initial position in the timber specimen, and bending deformation, considered as the relative deformation of the ends of the dowels with respect to the steel plate. In the case of 12 mm dowels, the bending deformation was larger than the crushing deformation, while it was smaller in the case of 20 mm dowels. Moreover, dowels loaded parallel to the grain showed larger bending deformations than dowels loaded perpendicular to the grain. They concluded that the loading of the individual dowels in the connection differs depending on their location.

Due to their increased risk of brittle failure in timber, the need to reinforce timber connections has been emphasized [5]. In addition to Awaludin et al. [18] and Bader et al. [19], several reinforcing methods have been proposed. Haller and Welshener [20] reinforced multiple dowelled timber-to-steel connections with glass fibre reinforced plastic (GFRP) as well as densified veneer wood (DVW). Blafsand Schadle [21] investigated the increase in both strength capacity and ductility of timber-to-timber connections with dowel-type fasteners reinforced by laterally inserted self-tapping screws. Moreover, engineered wood products, less prone to splitting, are increasingly used [3].

To correctly model the moment transmitting timber connections, due to their high importance in the global structural behaviour, the characterization of the wood is required. However, the particular features of timber material, such as its heterogeneity and its strong anisotropy, with different strengths in tension and compression and simultaneous ductile and brittle failure modes, lead to numerical problems and are a modelling challenge.

Sandhas et al. [22] classified and explained the different approaches that constitute the base to most of the existing numerical models for timber, and especially when applied to connections. Firstly, it is important to remember that ductile and brittle failure modes can occur in wood, and the existing approaches are based on specific methods for different problem cases. The identification of brittle failures usually leads to the choice of fracture mechanics approaches within a continuum framework or with discrete lattice models. These latter ones, in comparison to conventional continuum-based models, provide improved prediction of unconstrained crack initiation, propagation, evolution [23] and more realistic predictions of local-displacement response and experimentally observed damage patterns [24,25]. However, they are not simple to model by means of current finite element software.

The classical flow theory of plasticity in combination with a failure criterion (Hill criterion [26], Hoffman criterion [27] or the Tsai–Wu criterion [28]) have been generally used in continuum-based models. All previous, as phenomenological (and single-surface orthotropic) failure criteria, although able to accurately predict the failure onset, need additional modelling techniques to model the post-fracture response. Other modelling techniques, known as hybrid approaches, combine different constitutive laws to describe the mechanical behaviour of wood or to combine cohesive elements capable of representing splitting [29], but they are not an integral approach to model the 3D mechanical behaviour of wood.

Schweigler et al. [30] and Lemaitre et al. [11] developed and adapted engineering models based on nonlinear beam-on-foundation approaches to properly represent the global load–displacement curve of the timber connections, providing correct results for small and large displacement of up to 20 mm.

General multi-surface plasticity models were developed to identify different failure modes [31], but they are not yet suitable for applications where combinations of ductile and brittle failures may occur. Li et al. [28] adapted them to go beyond the elastic state and predict the strength of small clear wood.

Other models based on continuum damage mechanics (CDM) address the above-mentioned issues in one single 3D constitutive model and are able to describe the brittle failure modes in tension and shear as well as nonlinear elastic failure modes in compression [22].

This paper develops a Finite Element model for timber-to-timber beam-to-column moment transmitting connections. Not only must the geometrical and mechanical properties of the metal dowels and timber (different in both material directions) be taken into account, but also the contact phenomena and the fracture behaviour of timber. For these latter issues, the paper will explain in detail the contact stiffness parameter, as detailed in Iraola et al. [32], and a user subroutine based on Iraola and Cabrero [33] to model the timber failure and post-elastic response. The friction between timber elements was also studied, and considerations from Hirai et al. [34] were included to develop an accurate finite element model, capable of predicting the rotational stiffness and describe the load distribution among the dowels.

This paper is structured as follows: the current Section gives an overall introduction about the current work in the moment transmitting beam-to-column connections and the existing modelling techniques of such connections. Section 2 describes the materials and the experimental tests. The following sections focus on the FE model: Section 3 describes the methodology to create the FE model, Section 4 presents the analysis of the results, and Section 5 discusses the validation process of the different techniques and parameters used in the FE model, with the main conclusions eventually exposed in Section 6.

2. Experiments

2.1. Specimens

Specimens consisted of four different configurations of beam-to-column timber-to-timber moment transmitting connections. A typical specimen is shown in Fig. 1. The connection consisted of two parallel timber columns, classified as strength class C24 according to EN 338 [35] (where C refers to coniferous –softwood– timber, and 24 refers to its characteristic bending strength, 24 MPa), to which a central timber beam with the same strength class is attached by means of eight steel dowels. To obtain a tight fitting of the dowels, they were inserted in holes of the same diameter. The insertion process led to differences in the contact among the three timber members. As described below and investigated in this paper, some execution related aspects (i.e. manufacturing, tolerances, insertion and assembly) may affect the response of connections.

A rectangular pattern was chosen for the arrangement of the dowels, due to its simplicity for timber-to-timber connections and its higher moment capacity in comparison to a circular pattern [19].

The thicknesses of the timber elements were chosen based on the resulting yielding mode as described by the analytical approach based on
Johansen’s yield model and included in the current Eurocode 5 \cite{12,36}. By changing the thickness of the timber elements, the three different failure modes (pure embedment failure, yielding of one or three hinges in the fastener, Fig. 2) may happen. Table 1 shows the main dimensions of the four configurations and the principal failure mode expected for each one. It was intended to verify the differences among each resulting yielding mode, and to develop a finite element model capable of adequately distinguishing all of them. Furthermore, and due to the absence of reinforcement in the connection, brittle failure in wood was also expected. Fig. 3 shows the possible brittle failure modes that may occur in timber connections.

 Twelve tests were carried out in the University of Navarra, fixing the upper and downer column’s ends to 20 mm thick steel plates, which were pinned supported. Fig. 4 shows the structural layout of the cross section while the main dimensions are specified in Table 1. An Ibertest machine (397 kN total capacity) was used to apply a vertical load ($P$) at a distance of 100 mm from the right extreme of the beam. The load cycle according to EN 26891 \cite{38} was followed. The specimens were loaded up to 40% of the estimated ultimate load ($F_u$), maintained for 30 s and unloaded from $0.4F_u$ to $0.1F_u$. Then, the load was maintained at $0.1F_u$ for 30 s. In the final stage, the reloading started from $0.1F_u$ and continued until failure of the specimen. Superficial deformation and displacements of each test were analysed with the Digital Image Correlation Software GOM \cite{39}. A JAI Go-5000M-USB camera was employed, with an image frequency of 1 Hz.
Fig. 3. Possible failure modes of a timber connection with dowel-type fasteners: embedment is the only ductile failure mode, the rest are brittle [37].

Fig. 4. Specimen layout showing the main dimensions of the parts. Geometrical values given in Table 1.

Table 1
Experimental series. Geometry (in mm) and main failure modes expected (in one case, two possible failure modes are given, since the predicted load capacity for both modes differed less than 5%).

<table>
<thead>
<tr>
<th>series</th>
<th>Beam dimensions</th>
<th>Columns dimensions</th>
<th>Dowel diameter</th>
<th>Dowel spacing</th>
<th>Failure mode expected</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(h_b)</td>
<td>(b_b)</td>
<td>(c_b)</td>
<td>(h_c)</td>
<td>(b_c)</td>
</tr>
<tr>
<td>V_100</td>
<td>2600</td>
<td>200</td>
<td>100</td>
<td>1740</td>
<td>200</td>
</tr>
<tr>
<td>V_50</td>
<td>2600</td>
<td>200</td>
<td>50</td>
<td>1740</td>
<td>200</td>
</tr>
<tr>
<td>V_65</td>
<td>1700</td>
<td>130</td>
<td>65</td>
<td>1740</td>
<td>130</td>
</tr>
<tr>
<td>V_60</td>
<td>2600</td>
<td>200</td>
<td>60</td>
<td>1740</td>
<td>200</td>
</tr>
</tbody>
</table>

2.2. Materials

The selected wood species was Spruce (Picea abies (L.)), and all the timber members were classified as strength class C24 according to EN 338 [35]. The density of the used timber pieces was measured according to standard EN 408 [40]. The moisture content was measured with a thermo-hygrometer, being 12.7% the average of all pieces. The experimental values from Iraola [41] were used in the FE model, and all of them are shown in Table 2.

In order to characterize the graded as AISI 304L [44] stainless steel dowels, three tensile tests for each dowel diameter were carried out, following the UNE 7-474-92 [43]. As referred by other works [45], an over-strength was experimentally found, which may lead to problems when designing connections to dissipate energy, i.e. in seismic regions.
The main mechanical characteristics are shown in Table 3, and those obtained experimentally were adopted in the FE model.

### 3. Finite Element Model (FEM)

The Abaqus software [46] was used for the development of the FE model. It is widely used for modelling and analysis of mechanical components and assemblies and timber connections [45,47]. Within this section, a description of the used model parameters is given. Further discussion and validation details are given in Section 5.

#### 3.1. Geometry and boundary conditions

The timber-to-timber beam-to-column connection was modelled with one plane of symmetry to reduce the required computational cost. A Python parametric script was written to model each configuration to reduce the invested time in the modelling process. The parametric script depends on a text file where all geometrical variables described in Table 1 are defined.

As shown in Fig. 5, the displacements of both extremes of the column are constrained, while the rotations are allowed to reproduce the experimental boundaries. A reference point was used at both column’s ends, whose surfaces were previously defined as rigid bodies, to apply the boundary conditions.

#### 3.2. Materials

##### 3.2.1. Timber beam and column

An elastic and orthotropic material model was defined following the methodology described in [33] to model progressive failure in wood and implemented by a user-defined USDFLD subroutine (User Defined Field, which allows to define field variables at a material point, and is therefore mesh-dependent). The used mechanical properties of the spruce wood are shown in Table 2, and Table 4 shows the Elastic modulus $E$, Poisson’s ratio $v$ and shear modulus $G$ adopted in the FE model. These last values were assigned to the timber elements (beam and columns) according to their orientations, as shown in Fig. 5. The mechanical properties in the two directions perpendicular to the grain (tangential and radial directions) were assumed to be identical [48].

#### 3.3. Finite element mesh

All components of the modelled connection were constituted by 8-node solid elements with reduced integration and hourglass control, using the solid element C3D8R. Geometry and mesh size have a high impact on the obtained FE results, since the user subroutine modifies the properties of each element, and it is therefore mesh-dependent. However, the initial stiffness of the connection is independent of the mesh size, according to Hassanieh et al. [48] and Iraola and Cabrero [33]. Considering this, and with the aim of obtaining the most homogeneous element distribution close to the contact zone, several strategies were followed:

- Partitioning the parts in the contact zone to control the resulting mesh.
- As the user subroutine considers the damage for complete elements, a fine mesh obtains an improved accuracy [33].
- The mesh is defined to ensure that the nodes of the different parts concur in the beginning of the analysis, to avoid contact problems between the different parts.

Fig. 17(b) shows the mesh in the dowel area as a result of the previous meshing considerations. The global mesh size was set to 9 mm for the timber elements and 0.5 mm for the steel part. A sensitivity analysis and discussion on the selected mesh size is provided in Section 5.2.

### Tables

#### Table 2

| Bending strength $[N/mm^2]$ | $f_{u,k}$ | 24 |
| Tensile strength parallel to grain $[N/mm^2]$ | $f_{u,a}$ | 14 |
| Tensile strength perpendicular to grain $[N/mm^2]$ | $f_{u,b}$ | 0.4 |
| Compressive strength parallel to grain $[N/mm^2]$ | $f_{c,a}$ | 21 |
| Compressive strength perpendicular to grain $[N/mm^2]$ | $f_{c,b}$ | 2.5 |
| Shear strength $[N/mm^2]$ | $f_{s,k}$ | 2.5 |
| Modulus of elasticity parallel to grain $[N/mm^2]$ | $E_{a}$ | 11600 |
| Modulus of elasticity perpendicular to grain $[N/mm^2]$ | $E_{b}$ | 370 |
| Shear modulus $[N/mm^2]$ | $G_{a}$ | 650 |
| Mean density $[kg/m^3]$ | $\rho_{a}$ | 420 |

$^a$Values obtained in this experimental campaign.

#### Table 3

| Diameter [mm] | d |
| 8 | 12 |
| 8 | 12 |

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield tensile strength $[N/mm^2]$</td>
<td>$f_{y,k}$</td>
</tr>
<tr>
<td>Ultimate tensile strength $[N/mm^2]$</td>
<td>$f_{u,k}$</td>
</tr>
<tr>
<td>Modulus of elasticity $[N/mm^2]$</td>
<td>$E_a$</td>
</tr>
</tbody>
</table>

#### Table 4

| Elastic modulus [MPa] | Poisson’s ratio | Shear modulus [MPa] |
| 11600 | 370 | 370 |

$^b$Values extracted from Iraola [41].
3.4. Contact

Contact between the connection parts is a decisive issue in modelling beam-to-column connections with dowel-type fasteners. In the validated FE model, surface-to-surface discretization method was used [46] and characterized in the normal and tangential directions. Two different contact conditions were imposed: between stainless steel dowels and wood specimens, and between columns and beam.

The contact behaviour in the normal direction for the timber-to-timber interface (Fig. 6(a)) was modelled with the “Hard Contact” model, and with the additional option “Allow separation after contact”. The studied experimental set-up applies mostly a frictional behaviour for the timber-to-timber contact and the contact behaviour in the normal direction bears a great influence on the results, as explained in Section 5.4.

In the case of the steel-to-timber contact (Fig. 6(b)), a linear pressure-overclosure relationship was defined, with a slope \( k \) (corresponding to the contact stiffness) as proposed by Iraola et al. [32], and whose value depends on the contact area:

\[
k = \frac{2600}{\sqrt{A}},
\]

being \( A \) the contact area between the dowel and timber element, which was assumed in this work as

\[
A = d \times t,
\]

where \( d \) is the dowel diameter and \( t \) the thickness of the timber element. Used values for contact stiffness are shown in Table 5, and further discussion on the topic is provided in Section 5.3.

As indicated by Dorn et al. [47], in the timber-to-steel contact, due to their difference in stiffness, the stainless steel surface was chosen as the master surface, which is allowed to penetrate to the slave surface (the timber surface). In the case of timber-to-timber contact, the column surface was defined as the master surface, while the beam section was defined as the slave surface.

In the tangential direction, a penalty friction formulation was incorporated for both contact types. The assumed coefficients of friction were 0.3 for the steel-to-timber [34] and 1.0 for the timber-to-timber contact [49]. Discussion on the values is given in Section 5.4.1.

The existing friction was related to the installation procedure and showed a high influence on the resultant stiffness. To model it, a

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam-Dowels contact stiffness ( k_{\text{beam}} ) [N/mm]</th>
<th>Column-Dowels contact stiffness ( k_{\text{column}} ) [N/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_100</td>
<td>91.92</td>
<td>91.92</td>
</tr>
<tr>
<td>V_50</td>
<td>130.00</td>
<td>130.00</td>
</tr>
<tr>
<td>V_60</td>
<td>96.90</td>
<td>75.06</td>
</tr>
<tr>
<td>V_65</td>
<td>114.02</td>
<td>114.02</td>
</tr>
</tbody>
</table>
normal pressure of 0.1N/mm² was applied in the connection area, on a predefined surface around the holes, as shown in Fig. 7. This surface $S_c$ was made dependent on geometrical variables, and defined by the region corresponding to the timber-to-timber contact interface (Fig. 6(a)) minus the surface corresponding to the holes of the dowels:

$$S_c = \left(2\left(a_1 + \frac{3}{2}d\right)^2 - 8\pi \left(\frac{d}{2}\right)^2\right)$$

Discussion on the required normal pressure and the selected value in the model is given in Section 5.4.

3.5 Analysis procedure

A displacement-controlled procedure was used to apply the load to the FE model, with a unitary defined displacement applied to the right edge of the beam where the load is applied at the experiment. A static Riks procedure was used.

The script used for the modelling of the fracture of timber [33] sequentially modifies and considers additional damage modes, leading to a sequential change of the material properties of each element according to its failure state, resulting in numerical instabilities. As a consequence, the increment size has a great impact on the simulation results. Both the number of iterations per increment $I_C$ and the number of attempts per increment $I_A$ were augmented from the default values ($I_C$, from 16 to 1000; $I_A$, from 5 to 30).

4. Results

4.1 Moment-rotation results

Four different configurations were tested experimentally (Table 1), considering three replicates for each arrangement to be able to consider the existing variability in the tests, which may be due to a variety of reasons, namely variation of material properties, or manufacturing and installation procedures.

The resulting moment-rotation curves are shown in Fig. 8. It can be seen that the numerical responses agree reasonably well with the experimental results, obtaining the best results for the $V_60$ and $V_65$ cases. Further comments on these results are provided in the following sections.

As the used subroutine to model fracture of timber considers the damage as happening in the whole element, and has not implemented any extended finite element method (XFEM), once the number of elements with damage in more than one direction increases, the high complexity of the model prevents the next step from convergence and consequently the model prematurely stops. This is the reason why the curves that correspond to FE model in Fig. 8 cannot further reproduce the post-failure behaviour. The current approach provides a good simulation of the elastic response of the connection, and furthermore, it is capable of detecting the transition between elastic and plastic regions.
4.2. Rotational stiffness

For each tested connection, the initial rotational stiffness was calculated in the loading phase according to Eq. (4):

\[ k = \frac{(M_2 - M_1)}{(\theta_2 - \theta_1)} \]

where \( M \) is the applied moment and \( \theta \) is the angle of rotation at 10% and 40% of the maximum applied moment for each specimen. The standard EN 26891 [38] considers the connection behaviour before the unloading phase to obtain the stiffness, and dismisses the initial loading range up to 10% of the maximum load to disregard possible adjustments.

The obtained values for each specimen are shown in Table 6. The rotational stiffness values vary from the lowest 22.95 kNm/\( \text{rad} \) for specimen V.65.1 to 124.07 kNm/\( \text{rad} \) for specimen V.60.3. The specimens V.100.1, V.65.3 and V.50.3 clearly differ from the average obtained values (see Fig. 8). Some possible causes for their different response could relate either to the variability in material properties or to the manual installation of the connection elements, which may result in a different imposed friction between the timber members. The specimen V.50.3 was intentionally assembled with the help of clamps, fixing beam and columns and, as expected, a stiffer response was observed in the obtained curves (see Fig. 8(b)).

4.3. Yield point estimation

Based on the discussion by Muñoz et al. [50], the so-called “5% of diameter” method has been used to obtain the yield point. It is defined
Table 6
Comparison between tested specimens responses and FE model results.

<table>
<thead>
<tr>
<th>Series</th>
<th>Symbol</th>
<th>Rotational Stiffness</th>
<th>Moment capacity</th>
<th>Load in Yield Point</th>
<th>Deformation in Yield Point</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Units</td>
<td>k KNm/rad</td>
<td>( M_{max} ) KNm</td>
<td>( F_{y} ) kN</td>
<td>( \sigma_{y} ) mm</td>
</tr>
<tr>
<td>V_100.1</td>
<td>28.76</td>
<td>3.53</td>
<td>1.66</td>
<td>64.00</td>
<td></td>
</tr>
<tr>
<td>V_100.2</td>
<td>65.74</td>
<td>3.38</td>
<td>2.12</td>
<td>62.50</td>
<td></td>
</tr>
<tr>
<td>V_100.3</td>
<td>66.91</td>
<td>4.34</td>
<td>2.17</td>
<td>62.01</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>53.80 (66.32)</td>
<td>3.75</td>
<td>1.98</td>
<td>62.84</td>
<td></td>
</tr>
<tr>
<td>Standard deviation (SD)^a</td>
<td>21.70 (0.83)</td>
<td>0.52</td>
<td>0.28</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>FE Model</td>
<td>80.69</td>
<td>2.89</td>
<td>2.11</td>
<td>52.61</td>
<td></td>
</tr>
<tr>
<td>FE Model/Average^a</td>
<td>150% (122%)</td>
<td>77%</td>
<td>106%</td>
<td>84%</td>
<td></td>
</tr>
<tr>
<td>V_50.1</td>
<td>47.99</td>
<td>2.38</td>
<td>1.68</td>
<td>68.70</td>
<td></td>
</tr>
<tr>
<td>V_50.2</td>
<td>47.02</td>
<td>2.44</td>
<td>1.87</td>
<td>70.49</td>
<td></td>
</tr>
<tr>
<td>V_50.3</td>
<td>66.30</td>
<td>3.28</td>
<td>1.96</td>
<td>62.00</td>
<td></td>
</tr>
<tr>
<td>Average^a</td>
<td>53.77 (47.50)</td>
<td>2.70</td>
<td>1.84</td>
<td>67.06</td>
<td></td>
</tr>
<tr>
<td>Standard deviation (SD)^a</td>
<td>10.86 (0.68)</td>
<td>0.50</td>
<td>0.14</td>
<td>4.48</td>
<td></td>
</tr>
<tr>
<td>FE Model</td>
<td>82.34</td>
<td>2.51</td>
<td>1.72</td>
<td>43.02</td>
<td></td>
</tr>
<tr>
<td>FE Model/Average^a</td>
<td>153% (173%)</td>
<td>93%</td>
<td>94%</td>
<td>64%</td>
<td></td>
</tr>
<tr>
<td>V_65.1</td>
<td>22.95</td>
<td>1.52</td>
<td>1.72</td>
<td>68.99</td>
<td></td>
</tr>
<tr>
<td>V_65.2</td>
<td>24.45</td>
<td>1.79</td>
<td>2.10</td>
<td>75.00</td>
<td></td>
</tr>
<tr>
<td>V_65.3</td>
<td>35.65</td>
<td>1.87</td>
<td>1.43</td>
<td>35.00</td>
<td></td>
</tr>
<tr>
<td>Average^a</td>
<td>27.68 (23.70)</td>
<td>1.73</td>
<td>1.75</td>
<td>59.66</td>
<td></td>
</tr>
<tr>
<td>Standard deviation (SD)^a</td>
<td>6.94 (3.06)</td>
<td>0.18</td>
<td>0.34</td>
<td>21.57</td>
<td></td>
</tr>
<tr>
<td>FE Model</td>
<td>24.99</td>
<td>1.20</td>
<td>1.23</td>
<td>45.54</td>
<td></td>
</tr>
<tr>
<td>FE Model/Average^a</td>
<td>90% (105%)</td>
<td>69%</td>
<td>70%</td>
<td>76%</td>
<td></td>
</tr>
<tr>
<td>V_60.1</td>
<td>110.88</td>
<td>5.01</td>
<td>3.91</td>
<td>68.39</td>
<td></td>
</tr>
<tr>
<td>V_60.2</td>
<td>104.69</td>
<td>4.05</td>
<td>2.71</td>
<td>54.50</td>
<td></td>
</tr>
<tr>
<td>V_60.3</td>
<td>124.07</td>
<td>3.99</td>
<td>2.88</td>
<td>50.00</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>113.21 (107.78)</td>
<td>4.35</td>
<td>3.17</td>
<td>57.63</td>
<td></td>
</tr>
<tr>
<td>Standard deviation (SD)</td>
<td>9.90 (4.38)</td>
<td>0.57</td>
<td>0.65</td>
<td>9.59</td>
<td></td>
</tr>
<tr>
<td>FE Model</td>
<td>113.19</td>
<td>3.94</td>
<td>2.52</td>
<td>44.93</td>
<td></td>
</tr>
<tr>
<td>FE Model/Average</td>
<td>102% (107%)</td>
<td>91%</td>
<td>80%</td>
<td>78%</td>
<td></td>
</tr>
</tbody>
</table>

^aValues given in brackets are obtained not considering the out-layer of each configuration (V_100.1, V_50.3, V_65.3 and V_60.3).

Fig. 9. Observed damage in beams of each configuration.
where only one principal failure mode was expected (mode k for V.100 configuration and mode j for V.50 configuration). A lower percentage of accuracy is obtained for the displacement.

All experiments had to be stopped when the maximum displacement capacity of the loading machine (close to 200mm) was achieved. Therefore, no data on the obtained experimental ductility is given.

### 4.4. Deformation and failure modes

Predicted failure modes and damage in wood (Figs. 2 and 3) show good agreement with the experimental results. The resulting SDV1 parameter given by the subroutine [33] shows the damage propagation in the analysed configurations, as shown in Fig. 10.

In the V.60 series, the experimentally observed longitudinal cracks on both connection corners in Fig. 9(d) are also visible in the model (Fig. 10(d)). The subroutine correctly classifies the resulting failure as splitting produced by tension in the direction perpendicular to the grain [33]. The same brittle tensile failure is visible in V.65 and V.50 series (Figs. 9(b) and 9(c)), as shown in Figs. 10(b) and 10(c).

Regarding the predicted yielding modes, V.60 series showed no yielding in the stainless steel dowels (mode h), since the previously described splitting occurred before yielding. V.50 series showed one plastic hinge in the dowels (mode j). Fig. 11(b) shows the plastic strain of the modelled dowel, and its qualitative comparison with the tested dowel. Detected failure in timber is again splitting. Two plastic hinges are produced in the dowels (mode k) of the V.100 configuration, as shown in Fig. 9(a), and as predicted by the FE model in Fig. 11(a). Due to the numerical instability produced by the material subroutine, the model prematurely stops and, although the deformation shape shows

---

**Fig. 10.** Predicted damage sequence in beam with the user subroutine (SDV1) for the simulation increments 10, 20 and 30, with the damaged elements represented in dark grey. The damaged elements in the direction perpendicular to the grain are represented in red. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

(a) V.100 configuration.  
(b) V.50 configuration.  
(c) V.65 configuration.  
(d) V.60 configuration.
the predicted two plastic hinges in the FE model, plastic strain is observed only in one location. The analytical approach assumed two possible failure modes (one or two plastic hinges in dowels—modes j and k) for the V_65 series. As shown in the predicted deformation of the dowel (see Fig. 11(c)), the response is an intermediate state between both.

4.5. Digital image correlation results

The Digital Image Correlation Software GOM [39] was used to analyse each test, allowing to compare the superficial major strain distribution along the beam and columns with the FE model. The major technical strains perpendicular to the grain (%) in the column for each specimen tested, which are concentrated in the vicinity of the dowels, were qualitatively studied. The measured highest strain values in the timber elements correspond to the zone where the loads in the dowels direct them to, as shown in Fig. 12. The load distribution among the dowels is discussed in more detail in the following Section 4.6.

The surface strain distribution along one side of the beam was also qualitatively studied. The localization of the beginning point of failure in wood is well predicted by the FE model, as supported by the investigations of cracks by the GOM software (see Figs. 10 and 13).

4.6. Load distribution in dowels

Load distribution among dowels is essential in timber connections. It directly relates to the existing dependence between loading direction and elastic deformations of the timber matrix. Moreover, it may be additionally affected by the anisotropy of timber, which may provoke deviations between the actual displacement and the force orientation [10].

The load distribution among dowels from the FE model is shown in Fig. 14 for each configuration, where the load transferred by each dowel was extracted from the reaction force of the point corresponding to the centre (r) of each dowel, situated in the plane of symmetry, and the total force from the reaction forces of the constrained edges of the columns. Based on the simplified analytical models [13,14], higher load values are expected for those situated at the corners (numbered as 1, 3, 5, and 7 in Fig. 14). However, and due to the actual combination of moment and shear loads, the obtained results show different tendencies.

Maximum approximate values for V_100 (Fig. 14(a)), V_50 (Fig. 14(b)) and V_65 (Fig. 14(c)) series correspond to dowel 5. For these three configurations, the loads in the lower dowels (5, 6, and 7) are the highest ones, indicating a displacement of the centre of rotation. In the case of V_100 series, in which no failure in timber elements was
appreciated, the loads at dowels 7 and 6 are quite similar. However, in the case of the V.50 and V.65 configurations, where splitting occurred, the load difference between that at dowel 5 and those at dowels 6 and 7 increases as the timber member splits with increasing load.

The trend is different in the case of the V.60 series, which presented splitting as well but no plastic hinges. Dowel 1 is the most loaded (Fig. 14(d)), to be followed by the dowels located in the lower row. However, the differences in load distribution are not as extreme as those in the other series.

The resulting direction of the load in the dowels is shown in Fig. 15. Angle \( \theta \) was obtained from the application of \( \theta = \arctan \frac{U_2}{U_1} \) (where \( U_1 \) and \( U_2 \) are the relative displacements in the \( X \) and \( Y \) directions of the previously mentioned centre, \( r \), of each dowel), and angle \( \phi \) from the moment–rotation curves. Three differentiated groups can be distinguished: mainly constant orientation of 90° at dowels 4 and 8 (middle row), and of 0° at dowels 2 and 6 (centre dowels in upper and lower rows), and in the range of 40–45° for dowels 1, 3, 5, and 7 (dowels in the corner). In this latter case, it is only kept constant in the V.100 configuration (Fig. 15(a)), that in which timber does not fail. When timber fails, as it is the case in V.50 (Fig. 15(b)), V.65 (Fig. 15(c)) and V.60 (Fig. 15(d)), the angle reduces from 10° to 20°. Due to the produced splitting, the dowels do not find any resistance to penetrate the wood, and tend to follow the path in the longitudinal direction of the wood. The predicted load directions show a good agreement with the location of the major strain values found by the Digital Image Correlation observations (Fig. 12).

5. Discussion on the different parameters of the finite element model

5.1. Modelling of timber fracture non-linear behaviour

Modelling of timber fracture and response is not considered within commercial finite element software packages. In this work, the implementation developed by Iraola and Cabrero [33], by means of a user defined USDFLD subroutine, was used.

Fig. 16 shows the influence of applying the material model subroutine in one of the configurations, V.60. Although irrelevant for the consideration of the initial stiffness, not only it provides an estimation of the load carrying capacity and a description of the resulting failure mode, but it serves to validate additional modelling procedures. As it will be shown in the following sections, the use of this material script, although it may yet lack some precision, is highly relevant for an adequate validation of the other parameters used in the model. For example, as described in Section 5.4, it allows to validate the required pressure responsible for an additional load transfer mechanism between the timber members, based on the resulting failure mode.
5.2. Mesh size

The used subroutine to model the non-linear response of timber [33] is sensitive to the mesh size. The script modifies the material parameters of the elements of the model according to the observed failure state, which may lead to numerical instability. As a general rule, the smaller the mesh size the more stable the model results are, and the more analysis steps the model will be able to perform. As a consequence, the maximum predicted load by the model depends on the mesh size, since it may relate to that step in which the model is not able to converge due to numerical instabilities.

Herein, the results from three different mesh sizes are discussed for the connection with 8 mm diameter dowels (V_65 configuration), whose obtained moment–rotation curves are shown in Fig. 17. All the mesh alternatives make use of the considerations described in Section 3.3, which were developed to avoid contact problems. Hence, described differences in this section are related only to the mesh size. The analysed configurations, shown in Figs. 17(a)–17(c), differ in the number of elements around the dowel hole, being 3 (coarse), 6 (medium) and 9 (fine). Table 7 compares the number of elements and the estimated amount of required RAM memory: the fine mesh size required 73 GB (more than the 64 GB available at the used workstation), doubling the required amount for the medium mesh size, and 18 times higher than for the coarse one.

Table 7

<table>
<thead>
<tr>
<th>Mesh size</th>
<th>Number of elements on seed edges</th>
<th>Problem size</th>
<th>Memory estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[Number of elements]</td>
<td>[Number of elements]</td>
<td>[MB]</td>
</tr>
<tr>
<td>3 (coarse)</td>
<td>116 195</td>
<td>4 657</td>
<td></td>
</tr>
<tr>
<td>6 (medium)</td>
<td>510 519</td>
<td>30 386</td>
<td></td>
</tr>
<tr>
<td>9 (fine)</td>
<td>1 024 595</td>
<td>73 627</td>
<td></td>
</tr>
</tbody>
</table>

Only the medium and coarse mesh sizes provided reasonable results (Fig. 17(d)), while the fine-mesh model was not able to converge, produced quite bad quality results, and showed numerical instability at a very low load rate (Fig. 17(e)). The medium-sized model stops before than the coarse-sized mesh (Fig. 17(d)), as the volume of damaged elements increases at a higher rate. In the presented work, the medium mesh size was chosen as a good compromise between accuracy and computational efficiency. It must be emphasized that, as shown in Fig. 17(d), the obtained initial stiffness of the connection is independent of the mesh size.

5.3. Stiffness contact value

The need for a reduced contact stiffness parameter was discussed in Iraola et al. [32], as well as by other researchers [45,51], who, in the case of axially loaded connections, showed how numerical finite element models overestimate the experimental stiffness. As explained in Section 3.4, a softened contact stiffness parameter [32] was introduced, and this work uses it for the first time in a model of moment-transmitting connections.
Fig. 18 shows the comparison of the applied softened parameter to the usual hard-contact model. As shown, not only the stiffness prediction improves when including the contact stiffness, but also the failure response of the connection is more closely predicted. The model with hard contact results in a premature failure which does not correspond to reality.

Nevertheless, this developed stiffness contact value has some limitations. As it is dependant on geometrical parameters (Eq. (1)), and was validated on rectangular axially loaded specimens, in some cases it might lead to overestimation of global stiffness values. This is the case of V_50, with the lowest thickness value and the highest differences between predicted and experimental rotational stiffness (Fig. 8(d) and Table 6).

Table 8 Comparison between tests stiffness (with and without dowels) and FE model rotational stiffness for the V_60 configuration.

<table>
<thead>
<tr>
<th></th>
<th>Experimental stiffness w/o dowel [KNm/rad]</th>
<th>Experimental stiffness with dowel [KNm/rad]</th>
<th>FE Model stiffness [KNm/rad]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SNDT</td>
<td>110.02</td>
<td>113.21</td>
<td>115.19</td>
</tr>
<tr>
<td>SDT</td>
<td>112.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SDM</td>
<td>114.86</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
\text{SNDT} / \text{SDT} \quad 97\%
\text{SNDT} / \text{SDM} \quad 96\%
\text{Standard deviation (SD)} \quad 2.61
\]

5.4. Friction between timber elements

During the experimental campaign, in the production of most of the specimens it was observed how for the insertion of the dowels a significant effort was required, and, as a result, a tight contact among
the timber elements was achieved. Those few individual specimens in which such phenomenon was not experienced resulted in a lower rotational stiffness, which may be observed in Fig. 8. Therefore, a normal pressure between the timber elements, previously defined as contact pressure by Almeida et al. [52] and McKenzie and Karpovich [53], and whose effects on the lateral resistance of connections has been emphasized by Hirai et al. [34] and Gečys et al. [54], was introduced in the model to activate such observed friction between the timber elements (see Table 8).

Although in design practice it is usual to dismiss this effect, as it is assumed to be conservative, it could not be disregarded for the validation process against experimental tests. A non-standard experimental verification of the occurrence of such friction was obtained by testing a specimen of the V_60 configuration, in which both beam and columns were fixed only by clamps, and without dowels (Fig. 19(a)). Fig. 19(b) shows how the initial stiffness of the connection without dowels was 110 kNm/ rad, quite similar to the experimental one obtained when dowels were connecting the timber members. A clear reduction of the capacity to 0.65 kNm was obtained, due to the fact that no dowels were placed.

5.4.1. Friction coefficient

The assumed friction coefficient \( \mu = 1.0 \) may be considered as too high in comparison with those used in other investigations, such as the ones developed by Hassanieh et al. [48] and Aira et al. [55]. As shown in Fig. 20, there is a clear influence of the chosen friction coefficient and the obtained response. It can be noticed that the influence of the coefficient of friction on the initial stiffness of the connection must be taken into account.
To determine an optimal combination between $\mu$ and the applied normal pressure ($P$), a parametric study was implemented for different combination values, with the coefficient of friction $0.3 \leq \mu \leq 1.0$ and the normal pressure value $0.08 \leq P \leq 0.26$. In all cases, the pressure value was modified to equalize the frictional force with the normal force:

$$F_f = \mu F_n$$

where $F_f$ is the frictional force and $F_n$ is the normal force, and $\mu$ is the coefficient of friction.

Obtained results are given in Fig. 20 for V_60 and V_100 configurations. Not only the stiffness varies for each friction coefficient–normal pressure combination, but also the load carrying capacity does so. In some cases, the applied pressure induces a premature failure of the timber elements due to compression perpendicular to grain, which does not happen in the test. Such is the case, for example for the combinations $\mu = 0.3$ and $P = 0.26 \text{ N/mm}^2$ (see Fig. 20(a)), and $\mu = 0.5$ and $P = 0.15 \text{ N/mm}^2$ (Fig. 20(b)) which, although lead to a value of initial stiffness close to the experimental one, obtain a premature failure in wood. Similar results were obtained for V_50 and V_65 configurations. For this reason, the combination of $\mu = 1$ and $P = 0.1 \text{ N/mm}^2$ was selected to model the connection.

Although it is usually recommended to dismiss this friction among connection elements, it was found necessary for an accurate simulation of the presented testing set-up. As shown in the case of the V_100 FE model, when no pressure is applied, the moment–rotation curve perfectly fits with that of the experiment in which no contact among timber elements was achieved.

6. Conclusions

In the present paper, a FE model for moment transmitting timber-to-timber beam-to-column connections was developed and analysed...
with Abaqus software. The properties of each material, the anisotropic behaviour of timber, and the non-linear behaviour of the connection area, were considered to improve the accuracy of the model.

A study of four different configurations of this type of connection was conducted to validate the FE model, showing a good agreement between the moment–rotation responses of the model and the experimental tests overall.

The contact stiffness as proposed by Iraola et al. [32], based on the geometry of the contact area, allows to obtain an improved prediction of the moment–rotation response of the connection for most of the configurations.

Although the friction between timber members and elements of the connection is neglected in design practice (as it is assumed a conservative assumption), its importance has been shown in this work. The installation procedure may develop contact pressure, which induces an additional rotational stiffness and should be considered for validation purposes. The model reproduced such friction by applying a normal pressure to the connection area.

The proposed model allows to differentiate between ductile and brittle failure modes. The analysis of rotational stiffness and yield point coordinates shows that the best correlations between the predicted and experimental rotational stiffness are obtained for the series V_60 and V_65 (whose failure modes were mode h for V_60 series and mode j for V_65, with splitting in both cases), with percentages from 90% (V_65) to 102% (V_60). For the rest of cases, the FE model overestimates the rotational stiffness values. Taking this into consideration, the FE model is capable of predicting more effectively the stiffness of moment-transmitting connections in which the failure of the wood is the principal one than those in which a ductile response is obtained.

The model allows to obtain a detailed analysis of the response of the connection, in terms of timber failure and load distribution. The obtained insight shows a load distribution pattern different to that usually assumed in simplified analytical models. Instead of those dowels in the corner, the dowels in the lower row result as the most loaded ones. Moreover, the changes in the load distribution and in the resulting load angle due to timber fracture are described.
Fig. 20. Influence of the combination of values $\mu$ and $P$ in the subroutine, compared with the selected $\mu = 1$ and $P = 0.1$ N/mm$^2$. (The lower the $\mu$, the higher the $P$, and consequently more elements will be early damaged because of the subroutine).

More studies should be done in order to optimize the developed model, with the aim to reduce the computational cost, which was not an indispensable objective in this model. Material models with improved numerical stability should be developed to improve the prediction of the yield point and the ultimate displacement.

**CRediT authorship contribution statement**

**M. Basterrechea-Árevalo**: Conceptualization, Methodology, Validation, Investigation, Writing – original draft, Formal analysis, Visualization. **J.M. Cabrero**: Conceptualization, Methodology, Validation, Writing – review & editing, Supervision, Formal analysis, Funding acquisition, Project administration. **B. Iraola**: Methodology, Validation, Investigation, Writing – review & editing, Visualization. **R. Goñi**: Resources, Writing – review & editing, Visualization.

**Declaration of competing interest**

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

**Acknowledgements**

The financial support provided by the Spanish Ministerio de Ciencia e Innovación and Fondo Europeo de Desarrollo Regional under contract BIA2016-80358-C2-1-P MINECO/FEDER UE and the Friends of the University of Navarra Association, Spain is gratefully acknowledged.

**References**

19

back ground paper prepared for the 61st Session of the FAO advisory committee on sustainable forest-based industries. 2020, April.


